KEY TO BORING LOGS

MAJOR TYPES			DESCRIPTION
COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW - Well graded gravels or gravel-sand mixtures GP - Poorly graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES	GM - Silty gravels, gravel-sand and silt mixtures GC - Clayey gravels, gravel-sand and clay mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW - Well graded sands, or gravelly sand mixtures SP - Poorly graded sands or gravelly sand mixtures
		SANDS WITH OVER 12 % FINES	SM - Silty sand, sand-silt mixtures SC - Clayey sand, sand-clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS		ML - Inorganic silt with low to medium plasticity CL - Inorganic clay with low to medium plasticity OL - Low plasticity organic silts and clays
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %		MH - Inorganic silt with high plasticity CH - Inorganic clay with high plasticity OH - Highly plastic organic silts and clays
	HIGHLY ORGANIC SOILS		PT - Peat and other highly organic soils

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE: 200, 40, 10, 4

CLEAR SQUARE SIEVE OPENINGS: 3/4 ", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

RELATIVE DENSITY

SANDS AND GRAVELS	BLOWS/FOOT (S.P.T.)
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS	STRENGTH*	BLOWS/FOOT (S.P.T.)
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-15
VERY STIFF	2-4	15-30
HARD	OVER 4	OVER 30

MOISTURE CONDITION

DRY	Absence of moisture, dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater
SATURATED	Below the water table

MINOR CONSTITUENT QUANTITIES (BY WEIGHT)

TRACE	Particles are present, but estimated to the less than 5%
SOME	5 to 15%
WITH	15 to 30%
........Y	30 to 50%

SAMPLER SYMBOLS

- Modified California (3" O.D.) sampler
- California (2.5" O.D.) sampler
- S.P.T. - Split spoon sampler
- Shelby Tube
- Continuous Core
- Bag Samples
- Grab Samples
- NR No Recovery

LINE TYPES

- Solid - Layer Break
- Dashed - Gradational or approximate layer break

GROUND-WATER SYMBOLS

- Groundwater level during drilling
- Stabilized groundwater level

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EXCELLENT SERVICE SINCE 1971

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

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LOG OF BORING 1-B4

PACIFICA, CA
7443.1.001.01

DATE DRILLED: October 12, 2006
HOLE DEPTH (FT): 36.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (FT-MSL): XXXXX ft.

LOGGED / REVIEWED BY: L. Damerell / JW
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Stem Auger
HAMMER TYPE: Rope & Pulley

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
0	0		ASPHALTIC CONCRETE						
			AGGREGATE BASE						
			SILTY CLAY (CH), dark brown, slightly moist, some sand, rootlets (FILL)						
			light brown, slightly moist, some sand (FILL)						
	1								
5			light brown, moist, medium stiff, gravel to 3/4in grading to						
			SILTY SAND (SM), light brown, moist, medium dense, medium grained (FILL)			20			
	2								
			Driller comment: gravel at this depth						
			SILTY CLAY (CL), mottled light brown and brown, slightly moist, stiff, gravel to 3/4in						
10	3					17			
	4								
15			Driller's comment: rocky at this depth.						
			SILTY CLAY (CL), tan, moist, hard, manganese staining (FILL)			34			
	5				▼				
20	6								
			gravel to 3/8in dia.			18			
			grayish brown, slightly moist, very stiff, fine to coarse sand, gravel to 1 1/2in dia.						
	7		Driller's comment: ground hard then soft						
25			mottled brown and tan, moist, hard, fine to coarse, slight manganese staining (FILL)			38			
	8								
	9				▽				
30									

10-18-2006 G:\Active Projects\7443\7443100101 RW Pipeline\Borelogs\7443-1-B4.bor

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

BORE LOG 1-B4
CCWRP PUMP STATION
PACIFICA, CALIFORNIA

PROJECT NO.:	7443.1.001.01
DATE:	NOVEMBER 2006
DRAWN BY: PC	CHECKED BY: JW
FIGURE NO.	4

G:\Drafting\DRAFTING2_Dwg\7443\001\Pump Station\7443100101-PumpSta-4-5-BoreLog-1206.dwg 12-05-06 01:24:07 PM

ORIGINAL FIGURE PRINTED IN COLOR

ENGEO INCORPORATED

LOG OF BORING 1-B4

PACIFICA, CA
7443.1.001.01

DATE DRILLED: October 12, 2006
HOLE DEPTH (FT): 36.5 ft.
HOLE DIAMETER: 4.0 in.
SURF ELEV (FT-MSL): XXXXX ft.

LOGGED / REVIEWED BY: L. Damerell / JW
DRILLING CONTRACTOR: West Coast Exploration
DRILLING METHOD: Solid Stem Auger
HAMMER TYPE: Rope & Pulley

Depth in Feet	Depth in Meters	Sample Type	DESCRIPTION	Log Symbol	Water Level	Blow Count / Foot	Moisture Content (% dry weight)	Dry Unit Weight (pcf)	Unconfined Strength (tsf) *field approx
30			SANDY SILTY CLAY (CL), tan, saturated, very stiff, fine to coarse (FILL)			25			
			moist						
	10								
35			GREENSTONE, reddish brown, plastic to friable, crushed, massive, deep			53/6			
	11								

BOTTOM OF BORING AT APPROXIMATELY 36.5 FT.
GROUNDWATER NOT ENCOUNTERED DURING DRILLING.

Depth scale: 40, 45, 50, 55, 60 ft; 12, 13, 14, 15, 16, 17, 18 m

10-18-2006 G:\Active Projects\7443\7443100101 RW Pipeline\Borelogs\7443-1-B4.bor

ENGEO INCORPORATED
EXCELLENT SERVICE SINCE 1971

BORE LOG 1-B4
CCWRP PUMP STATION
PACIFICA, CALIFORNIA

PROJECT NO.: 7443.1.001.01
DATE: NOVEMBER 2006
DRAWN BY: PC
CHECKED BY: JW
FIGURE NO. 5

G:\Drafting\DRAFTING2_Dwg\7443\001\Pump Station\7443100101-PumpSta-4-5-BoreLog-1206.dwg 12-05-06 01:24:27 PM

ORIGINAL FIGURE PRINTED IN COLOR

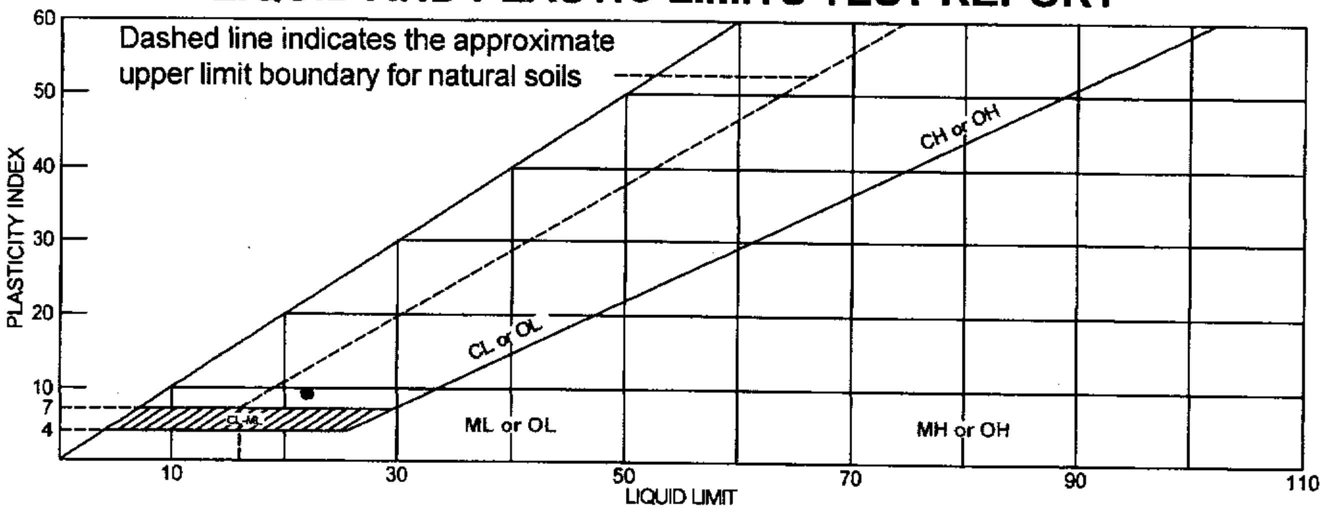
ENGEO
INCORPORATED

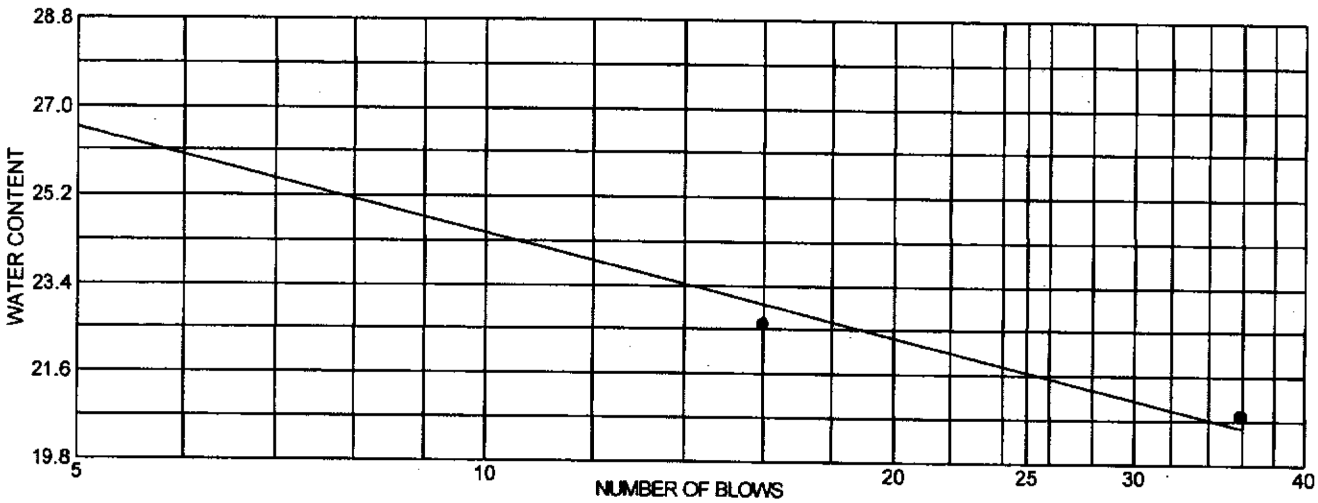
APPENDIX A

Laboratory Results

7443.1.001.01
November 3, 2006

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark brown clayey SAND to sandy CLAY	22	13	9			SC-CL

Project No. 7443.1.001.01 **Client:**

Project: NCCWD Recycled Water Project

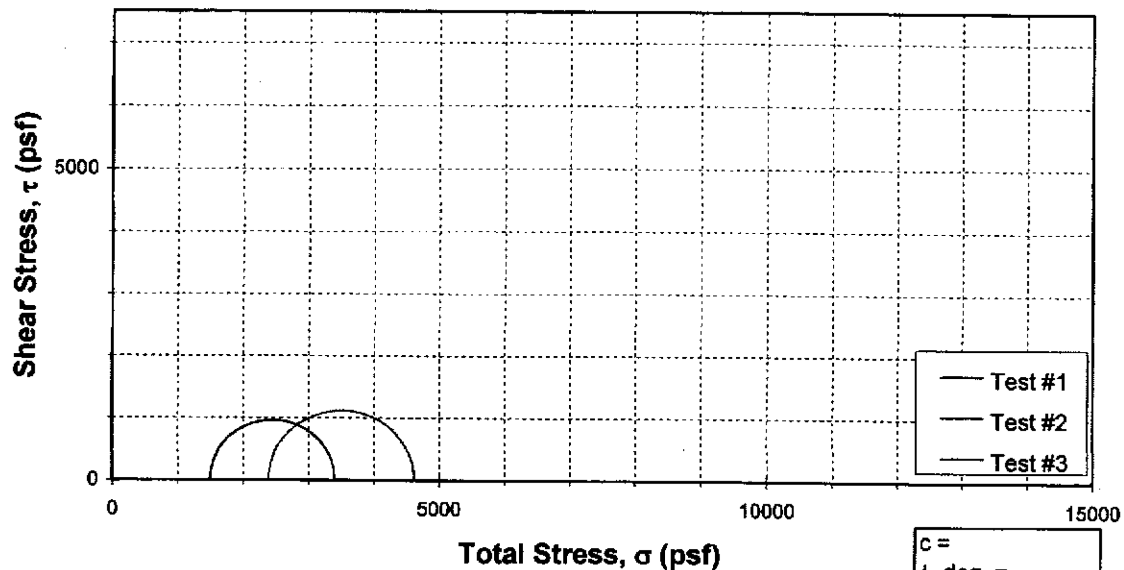
● **Source:** **Sample No.:** B4@5 (UPR)

Remarks:
● B4@5 (UPR)

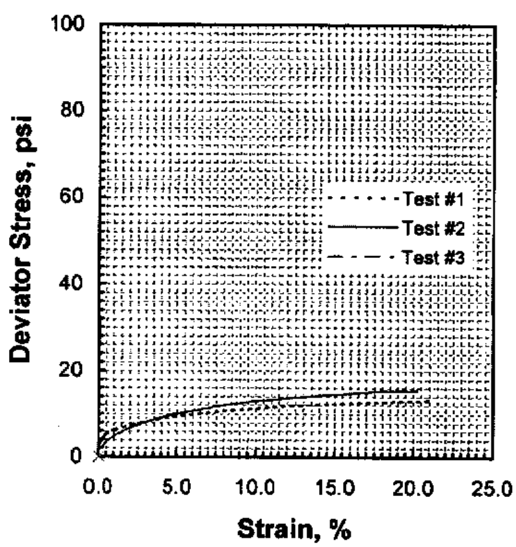
ENGEO INCORPORATED — GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS MATERIALS TESTING

TRIAXIAL COMPRESSION TEST REPORT

TRIAXIAL TEST - UNCONSOLIDATED UNDRAINED (UU)



c =
ϕ, deg. =
tan ϕ =



Test Data

Test No.	1	2	3	4
Initial				
Water Content, %	21%	17%		
Dry Density, pcf	109.0	117.2		
Saturation, %	99%	99%		
Void Ratio	0.60	0.49		
Minor Principal Stress, psf	1497.6	2400.5	0.0	0.0
Maximum Deviator Stress, psf	1910.0	2223.1	0.0	0.0
Time of Failure, min				
Rate of Strain Increments, %/min				
Initial Diameter, in	2.42	2.42	0.00	0.00
Initial Height, in	4.70	4.85	0.00	0.00
B- Value				

Test No.	Description of Specimens:	Sample No.	Sample Depth	LL	PI
1	Yellowish brown sandy silty CLAY with gravel	B4@15'	15'		
2	Yellowish brown sandy silty CLAY with gravel	B4@30'(2400psf)	30'		
3					
4					

Comments:		
	Boring Number:	
	Project Name:	NCCWD Recycled
	Project Number:	7443.1.001.01
	Technician:	DS
	Date:	

ENGEO
INCORPORATED

APPENDIX B

Corrosivity Analysis

7443.1.001.01
November 3, 2006

California State Certified Laboratory No.2153

CERCO
analytical, inc.

3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www cercoanalytical.com

15 November, 2006

Job No.0611100
Cust. No.10169

Ms. Kendra Maghoney
ENGEO Inc.
2010 Crow Canyon Place, Suite 250
San Ramon, CA 94583

Subject: Project No.: 7443.1.001.01
Project Name: NCCWD Recycled Water Project
Corrosivity Analysis – ASTM Test Methods

Dear Ms. Maghoney:

Pursuant to your request, CERCO Analytical has analyzed the soil samples submitted on November 10, 2006. Based on the analytical results, a brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurements, Sample No.002 is classified as "corrosive" and Sample No.001 is classified as "moderately corrosive". All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentrations range from 20 to 29 mg/kg. Because the chloride ion concentrations are less than 300 mg/kg, they are determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentrations range from 33 to 66 mg/kg and are determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at these locations.

The pH of the soils range from 7.9 to 8.1 which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potentials range from 410 to 420-mV, which are indicative of aerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call JDH Corrosion Consultants, Inc. at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.

J. Darby Howard, Jr., P.E.
President

JDH/jdl
Enclosure

California State Certified Laboratory No.2153

CERCO analytical, inc.
3942-A Valley Avenue
Pleasanton, CA 94566-4715
925.462.2771 • Fax: 925.462.2775
www.cercoanalytical.com

Client: ENGEO
Client's Project No.: 7443.1.001.01
Client's Project Name: NCCWD Recycled Water Project
Date Sampled: 10-Nov-06
Date Received: 10-Nov-06
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 15-Nov-2006

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
0611100-001	B4 @ 5'	410	8.1	-	3,900	-	20	66
0611100-002	B4 @ 25'	420	7.9	-	1,500	-	29	33

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Detection Limit:	-	-	10	-	50	15	15
Date Analyzed:	15-Nov-2006	14-Nov-2006	-	10-Nov-2006	-	14-Nov-2006	14-Nov-2006

* Results Reported on "As Received" Basis

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

NCCWD Pacifica Water Recycling Project
Environmental Assessment

APPENDIX C
Geotechnical Investigation NCCWD Gypsy Hill Tank

Land/Marine
Geotechnics
Geotechnical Consultants

Geotechnical Investigation
North Coast County Water District
Gypsy Hill Tank
Pacifica, California

Prepared for:
Kennedy/Jenks Consultants
Palo Alto, California

October 14, 2005
Project No. 105.005

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APPENDIX B
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APPENDIX C
Seismic Hazard Analyses

Author: Robert Pyke, Ph.D., G.E.

GEOTECHNICAL INVESTIGATION
NORTH COAST COUNTY WATER DISTRICT
GYPSY HILL WATER TANK
Pacifica, California

1.0 INTRODUCTION

This report presents the results of the geotechnical investigation performed by Land Marine Geotechnics for the proposed Gypsy Hill Water Tank Replacement Project located in Pacifica, California. The tank is owned and operated by the North Coast County Water District (District). The project site is shown on the Site Location Map, Figure 1 and Site Plan, Figure 2. Our services were provided in accordance with our proposal dated August 2, 2005.

2.0 PROJECT DESCRIPTION

The project site is located off Gypsy Hill Road near its intersection with Sharp Park Road in Pacifica, California. The site has been graded in the past to form a level pad and was previously occupied a 5-million-gallon steel water storage tank which was referred to as the Gypsy Hill Tank. This tank was recently demolished and the site is currently vacant. We understand the District plans to construct a new 3-million-gallon steel water storage tank within the footprint of the previous tank. The new tank will be located in the northern quadrant of the previous tank site. Some site grading including cutting a level pad into the slope along the northeast side of the site is planned. A new asphalt paved access road will be constructed to and around the tank.

3.0 SCOPE OF SERVICES

Our scope of services, as outlined in our proposal, consisted of exploring the subsurface conditions at the site and performing laboratory tests and engineering analyses, to develop conclusions and recommendations regarding:

- Soil, bedrock and groundwater conditions at the site

- Site seismicity and seismic hazards
- Site specific response spectra
- The most appropriate foundation type for the proposed tank including:
    - Allowable bearing pressures and lateral earth pressure coefficients for ring wall design
    - Soil unit weights for use in ring wall design
- Design criteria for the recommended foundation type, including vertical and lateral capacities
- Estimated foundation settlement
- Lateral pressures for retaining wall design, if required
- Site grading and subgrade preparation, including:
    - Fill quality and compaction requirements
    - Cut and fill slope inclinations
- 2001 California Building Code soil profile type and near-source factors
- Subgrade and base preparation for pavements
- Excavation characteristics of rock material
- Construction considerations

4.0 FIELD INVESTIGATION

Subsurface conditions at the site were explored by drilling five test borings at the approximate locations shown on Figure 2. The borings were drilled on September 2, 2005 using a truck-mounted solid flight auger drill rig. The borings extended 13 to 17.5 feet below ground surface (bgs). During drilling, our field engineer logged the soils encountered and obtained samples for visual classification and laboratory testing. The logs of the borings are presented in Appendix A on Figures A-1 through A-5. The soils encountered are classified according to the soil classification system described on Figure A-6. Rock was described in accordance with the criteria presented on Figure A-7.

Soil samples were obtained using two different types of samplers as presented below in Table 1.

TABLE 1
Sampler Details

Sampler Type	Outside Diameter (in)	Inside Diameter (in)	Liners
Modified California (Mod Cal)	3.0	2.5	Yes
Standard Penetration Test (SPT)	2.0	1.5	No

Resistance blow counts were obtained with the samplers by dropping a 140-pound hammer through a 30-inch free fall. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring log represent the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered. When the split spoon sampler was used, these blow counts are the standard penetration resistance values (N values). However, due to the large diameter of the Modified California sampler, the blow counts recorded for this sampler are not standard penetration resistance values. Approximate equivalent N values are presented on our log. These values were determined using a conversion factor of 0.6 for the Modified California Sampler.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and location indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

After completion, the borings were backfilled with cement grout. The soil cuttings were left onsite.

5.0 LABORATORY TESTING

We re-examined soil samples from the borings in our office to confirm field classifications and selected representative soil samples for testing. Selected samples were tested to measure moisture content, dry density, Atterberg Limits, and unconfined shear strength. Due to the significant percentage of rock fragments and gravel encountered in the samples, most of the samples were not appropriate for strength testing. The laboratory test results are presented on the boring logs and in Appendix B on Figures B-1 and B-2.

6.0 SITE CONDITIONS

6.1 Surface Conditions

We evaluated site conditions based on air photo interpretation and a site geological reconnaissance performed on September 13, 2005. The original steel water tank had been removed prior to our reconnaissance and grading near the existing gate had been completed. To evaluate the site history, 10 stereo aerial photograph pairs, dated 1949 through 2000, were reviewed from Pacific Aerial Surveys, Oakland and the U.S. Geological Survey library, Menlo Park. The date, scale, and photograph identification number for each of the photographs viewed are listed in the References section at the end of this report. Standard aerial photograph interpretation techniques were used to identify site conditions associated with erosion, and faulting, as well as identification of past site development or grading activities. These include mapping of tonal contrasts, arcuate or linear scarps or other abrupt changes in slope angle, lineations, and anomalous vegetation and drainage patterns.

The site encompasses a bench cut at Elevation 404 (North Coast County Water District, undated) approximately 50 feet below the crest of an east-west striking ridge. The materials mapped on the site include bedrock overlain by colluvium and fill. The tank pad has been cut into Franciscan Complex sandstone and fill has been placed to level the bench. The tank appears to have been constructed between the 1946 and 1955 aerial photographs. Approximately half of the original tank footprint was on fill along the southwest and northeast sides (Figure 2). Fill near the gate at the northeast corner of the site has been placed over colluvium in a pre-existing drainage swale. There is no evidence of recent slope instability. However, we noted erosion on the slope below the southwest side of the tank on the 1969 and 1977 aerial photographs.

6.2 Subsurface Conditions

The site was explored with four test borings advanced to depths of between 13 feet to 17.5 feet below the existing ground surface. Drain rock consisting of ¾ to 1 inch gravel was encountered at all the boring locations to depths of 9 to 12 inches. Test boring B-1 located on the north side of the tank pad encountered a foot of gravel fill overlying 7 feet of interbedded stiff clay and dense clayey sand that represents completely weathered claystone and sandstone of the Franciscan Complex. Siltstone interbedded with claystone was encountered to the bottom of the boring at 15.5 feet depth. The bedrock was intensely fractured, of low hardness, friable to weak and deeply weathered.

In test boring B-2, located in the approximate center of the tank pad, we encountered 3 feet of very stiff sandy clay below the gravel fill. Claystone bedrock was encountered at a depth of 4 feet to the bottom of the boring at 14.5 feet.

In test boring B-3, located at the southern side of the tank pad we encountered 3 feet of very stiff gravelly sandy clay below the gravel fill. Siltstone bedrock was encountered from 4 feet to the bottom of the boring at 13 feet deep.

In test boring B-4, located at the east side of the tank pad adjacent to the entrance road, we encountered 14 feet of colluvium overlying completely weathered claystone bedrock at depth of 14 feet to 17.5 feet at the bottom of the boring. The colluvium consisted of stiff to very stiff sandy clay with some sand and gravel fragments up to 3/4 –inch diameter.

In test boring B-5, located at the west side of the tank pad we encountered 4 feet of colluvium under the gravel fill. The colluvium consisted of stiff mottled clay with gravel up to 1-inch in diameter. Siltstone bedrock was encountered from 5 feet to the bottom of the boring at 16 feet below the ground surface.

Ground water was not encountered during drilling and sampling of these borings.

7.0 REGIONAL GEOLOGY AND SEISMICITY

7.1 Regional Geology

The site is located in the Coast Ranges geomorphic province that is characterized by northwest-southeast trending valleys and ridges. These are controlled by folds and faults that resulted from the collision of the Farallon and North American plates and subsequent shearing along the San Andreas fault. Bedrock in the region is primarily comprised of Upper Jurassic to Lower Cretaceous (~160-100 million years ago) Franciscan Complex rocks consisting of sandstone, shale, chert, greenstone, and localized limestone overlain by Quaternary alluvium and colluvium or ravine fill (Bonilla, 1971).

7.2 Seismicity

The major active fault in the area is the San Andreas Fault that is located less than 2 kilometers east of the site. However, there are several other active faults located in the region. For each of the active faults within 50 kilometers (km) of the site, the distance from the site and estimated maximum Moment magnitude[1,2] events are summarized in Table 1.

---

1 Moment magnitude is an energy-based scale and provides a physically meaningful measure of the size of a faulting event. Moment magnitude is directly related to average slip and fault rupture area.

2 California Division of Mines and Geology, 1996, *Probabilistic Seismic Hazard Assessment for the State of California*, CDMG Open-File Report 96-08.

TABLE 1
Regional Faults and Seismicity

Fault Name	Distance (km)	Direction from Site	Maximum Moment Magnitude
San Andreas - 1906 Rupture	1.7	Northeast	7.9
San Andreas - Peninsula	1.7	Northeast	7.2
San Gregorio North	6	West	7.3
San Andreas - North Coast South	23	Northwest	7.5
Monte Vista	29	Southeast	6.8
Hayward - Total	32	Northeast	7.1
Southern Hayward	32	Northeast	6.9
Northern Hayward	32	Northeast	6.6
Northern Calaveras	46	Northeast	7.0
Point Reyes	47	Northwest	6.8
Mount Diablo Thrust	47	Northeast	6.7

In 2002, the Working Group on California Earthquake Probabilities at the U.S. Geologic Survey (USGS) predicted a 62 percent probability of a magnitude 6.7 or greater earthquake occurring in the San Francisco Bay Area by the year 2030[3]. Smaller earthquakes (between magnitudes 6.0 and 6.7), capable of considerable damage if they occur in proximity to urban areas, have about an 80 percent chance of occurring in the Bay Area by 2032.

8.0 DISCUSSION AND CONCLUSIONS

We conclude that the proposed development is feasible from a geotechnical standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations regarding the project are discussed in the following sections.

3 Working Group on California Earthquake Probabilities (WGCEP), 2002, *Earthquake Probabilities in the San Francisco Bay region; 2000 to 2032 – A Summary of Findings*, Open File Report 99-517.

8.1 Seismic Hazards

During a major earthquake on a segment of one of the nearby faults, strong to very strong shaking is expected to occur at the site. Strong shaking during an earthquake can result in ground failure such as that associated with soil liquefaction[4], lateral spreading[5], and cyclic densification[6]. We used data from the test boring to evaluate the potential for these phenomena to occur at the site. The results of our evaluation are presented below.

8.1.1 Soil Liquefaction and Associated Hazards

We evaluated the liquefaction potential of soil layers encountered in our boring and concluded that they are not susceptible to liquefaction. Bedrock exists at shallow depth and the overlying soils are generally stiff clays. In addition the site is located at the top of a hill and groundwater was not encountered within the test borings. As a result, we conclude the potential for lateral spreading and for sand boils and lurch cracking at the ground surface are nil.

8.1.2 Cyclic Densification

Seismically induced compaction or cyclic densification of non-saturated sand (sand above the groundwater table) due to earthquake vibrations can result in settlement of the ground surface. Our field investigation indicates that the soil above groundwater is predominately stiff to very stiff clay. Therefore, we estimate the potential for ground surface settlement due to cyclic densification is nil.

---

4 Liquefaction is a transformation of soil from a solid to a liquefied state during which saturated soil temporarily loses strength resulting from the buildup of excess pore water pressure, especially during earthquake-induced cyclic loading. Soil susceptible to liquefaction includes loose to medium dense sand and gravel, and low-plasticity silt deposits.

5 Lateral spreading is a phenomenon in which surficial soil displaces along a shear zone that has formed within an underlying liquefied layer. Upon reaching mobilization, the surficial blocks are transported down slope or in the direction of a free face by earthquake and gravitational forces.

6 Cyclic densification is a phenomenon in which non-saturated, cohesion-less soil is compacted by earthquake vibrations, causing differential settlement.

8.1.3 Fault Rupture

Historically, ground surface displacements closely follow the trace of geologically young faults. The site is not within an Earthquake Fault Zone, as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no known active or potentially active faults exist on the site. In a seismically active area, the remote possibility exists for future faulting in areas where no faults previously existed; however, we conclude the risk of surface faulting and consequent secondary ground failure is very low.

8.2 Site Preparation and Tank Foundation

The site is currently blanketed by a layer of drain rock which consists of ¾ to 1 inch diameter gravel. The drain rock layer is about ¾ to 1 foot thick is underlain by stiff to very stiff clays and bedrock. The drain rock is suitable for reuse as a base for the new tank provided that the rock is densified using a vibratory drum compactor prior to tank construction. If grades need to be raised additional drain rock can be added following compaction of the existing layer. It is anticipated that the soil and rock at the site can be excavated with a conventional backhoe or excavator.

The tank shell will be supported on a concrete ring foundation. The ring wall foundation should extend at least 12 inches into the clay soils below the drain rock layer. The ring wall foundation should be designed using the criteria presented in Section 9.2 of this report.

The tank will be underlain by stiff clayey soils and bedrock which has moderate to low compressibility, respectively under the anticipated tank loads. The clayey fill and colluvial soils which underlie the west side of the tank pad are stiff to very stiff and have been consolidated under the load of the existing tank for many years. As a result, we anticipate that future site settlement under the new tank loads will be small i.e. less than 1 inch. No special measures are recommended to mitigate tank settlement.

9.0 RECOMMENDATIONS

9.1 Site Grading

9.1.1 Tank Pad Preparation

The existing drain rock surface beneath the proposed tank should be graded to form a smooth surface, sprinkled with water to moisten the drain rock and underlying subgrade, as directed by the geotechnical engineer and compacted with a large vibratory steel smooth drum roller. The roller should have a operating weight of at least 12,000 pounds and deliver a centrifugal force of at least 18,000 pounds per drum. The roller should make at least 4 passes over the entire tank area.

9.1.2 Fill Pavement Subgrade Preparation

In areas where asphalt pavement is planned the drain rock should be removed or be thoroughly blended with onsite clayey soil to such that the subgrade soils have at least 20 percent passing a number 200 sieve. The subgrade on which fill will be place should be scarified to a depth of at least 8 inches, moisture conditioned to about optimum moisture content and be compacted to at least 90 percent relative compaction[7]. The subgrade beneath pavements should be compacted to at least 95 percent relative compaction.

9.1.3 Fill Placement

Onsite or import fill, to be used as general site fill, should be moisture-conditioned to about optimum moisture content, placed in lifts not exceeding eight inches in loose thickness, and compacted to at least 90 percent relative compaction. Import fill if required should consist of soil that has a liquid limit less than 40 and plasticity index (PI) less than 12, and be approved by the geotechnical engineer. All fill placed at the site should be free of organic matter and contain no rocks or lumps larger than three inches in greatest dimension.

[7] Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by the ASTM D1557-91 laboratory compaction procedure.

9.1.4 Utility Trench Backfill

Excavations for utility trenches can be readily made with a backhoe. All trenches should conform to the current CAL-OSHA requirements.

Backfill for utility trenches and other excavations is also considered fill, and should be placed and compacted according to the recommendations previously presented. If imported clean sand or gravel (defined as soil with less than 10 percent fines) is used as backfill, it should be compacted to at least 95 percent relative compaction. Jetting of trench backfill should not be permitted. Special care should be taken when backfilling utility trenches in pavement areas. Poor compaction may cause excessive settlements, resulting in damage to the pavement section.

Where utility trenches enter the tank pad and pavement areas, an impermeable plug consisting of lean concrete or compacted clay soil, at least five feet in length, should be installed. Furthermore, where sand- or gravel-backfilled trenches cross undeveloped or landscaped areas and pass below asphalt or concrete pavements, a similar plug should be placed at the edge of the pavement. The purpose of these recommendations is to reduce the potential for water to become trapped in trenches beneath the tank or pavements.

9.1.5 Cut and Fill Slopes

We recommend cut and fill slopes not be steeper than 2:1 (horizontal to vertical). To protect against slope erosion, we recommend concrete-lined drainage ditches be placed at the top of all slopes (cut and fill) higher than 10 feet. The drainage ditches should flow into non-perforated pipes leading to suitable discharge facilities. The drainage ditches will require periodic cleaning of any debris or soil. We recommend protecting the fill slope against erosion by seeding or hydro-mulching with deeply rooted, fast growing vegetation. To protect the slope until the ground cover has germinated, jute matting or spray-type mulch should be placed on the slope.

9.1.6 Surface Drainage

Drainage control design should include provisions for positive surface gradients so that surface runoff is not permitted to pond, particularly above slopes or adjacent to tank foundations, roadways, pavements, or slabs. Surface runoff should be directed away from slopes and foundations and collected in lined ditches or drainage swales. The water collected should be directed to a storm drain or paved roadway. Discharge from the tank roof should be included in the collection system and not allowed to infiltrate the subsurface near the structure or in the vicinity of slopes.

9.2 Ring Wall Foundation

The tanks should be supported on a ring wall spread footing foundation. The continuous ring wall footing should be at least 18-inches wide. If isolated spread footings are used to support interior columns they should be at least 24-inches wide. Footings should extend at least 12-inches below the lowest adjacent soil subgrade (defined as the bottom of the gravel layer beneath the tank). The footings may be designed using allowable bearing pressures of 3,500 pounds per square foot (psf) for dead plus live loads and 4,500 psf for total loads, including wind or seismic forces. These values include factors of safety of at least 2.0 and 1.5 for dead plus live loads and total loads, respectively.

The ring wall foundation will be subjected hoop stress due to lateral earth pressures acting on the inside face of the footings. The soil with footings can be assumed to have a total unit weight of 125 pounds per square foot. A lateral earth pressure coefficient of 0.45 should be used to evaluate the lateral pressure due to the tanks surcharge of the footings.

Lateral loads can be resisted by a combination of passive pressure acting on the vertical faces of the footings and friction along the bases of the footings. Passive resistance may be calculated using an equivalent fluid weight (triangular distribution) of 350 pounds per cubic foot (pcf). The upper one-foot of soil should be ignored unless it is confined by slabs or pavement. Frictional resistance should be computed using a base friction coefficient of 0.40. These values include a factor of safety of about 1.5. Footings located adjacent to utility trenches should bear below an imaginary 1.5:1 (horizontal to vertical) plane projected upward from the bottom edge of the adjacent trench.

The footing excavations should be free of standing water, debris, and disturbed materials prior to placing concrete. We should check foundation excavations prior to placement of reinforcing steel to confirm suitable bearing material is present. We should recheck the condition of the excavations just prior to concrete placement to confirm the excavations are sufficiently moist.

9.3 Retaining Walls

For cantilever walls retaining with level backfill, we recommend designing the walls for active lateral pressures corresponding to an equivalent fluid unit weight of 40 pounds per cubic foot (pcf). Walls that are restrained from rotation at the top should be designed using at-rest pressures corresponding to an equivalent fluid unit weight of 60 pcf. Where traffic is expected within a distance equal to the height of the walls, the walls should be designed for an additional uniform lateral pressure of 100 psf to be applied over the entire height of the wall or 10 feet, whichever is less.

Because the site is in a seismically active area, the design should be checked for seismic condition, in which the wall pressure is determined by adding the earth pressure due to earthquake shaking to the active earth pressure. The incremental seismic pressure is approximated by a uniform pressure, in psf, of 12 times the height of the wall in feet. The lateral earth pressures recommended are retaining walls that are back drained to prevent the buildup of hydrostatic pressure. The retaining wall foundation should be designed using the criteria presented in section 9.2

9.4 Asphalt Pavements

The State of California flexible pavement design method was used to develop the recommended asphalt concrete pavement sections. We expect the final soil subgrade in asphalt-paved areas will generally consist of silty clay. R-value tests were not performed for the site. On the basis of our experience with this soil type, we selected an R-value of 5 for design. Traffic data are not available for the proposed access roadways.

Therefore, we have assumed traffic indices (TIs) of 4.0 and 5.0. Recommended pavement sections for these traffic indices are presented in Table 4.

TABLE 4
Pavement Section Design

TI	Asphaltic Concrete (inches)	Class 2 Aggregate Base R = 78 (inches)
4.0	2.5	6
5.0	4.0	9

(Note: The minimum thickness of asphalt concrete and aggregate base is 2.5 and 6 inches, respectively.)

Pavement components should conform to the current Caltrans Standard Specifications. The upper six inches of the soil subgrade in pavement areas should be moisture-conditioned to about optimum and compacted to at least 95 percent relative compaction and rolled to provide a smooth non-yielding surface. Aggregate base should be compacted to at least 95 percent relative compaction.

9.5 Seismic Design

The San Andreas Fault is located about 1.7 miles north east of the site is the most significant seismic source potentially generating strong ground motion at the site. A Probabilistic Seismic Hazard Analysis (PSHA) was conducted for site by Dr. Robert Pyke, the results of which are presented in Appendix C. While the response spectra in the PSHA takes precedence for purposed of comparison seismic design in accordance with the 2001 California Building Code we recommend using the following parameters:

- Seismic Zone Factor 4; Z= 0.4
- Soil Profile Type S_{B-C}

- Near Source Factors N_a and N_v of 1.5 and 2.0, respectively.

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10.0 ADDITIONAL GEOTECHNICAL SERVICES

Prior to construction, Land Marine Geotechnics should review the project plans and specifications to check their conformance with the intent of our recommendations. During construction, our field engineer should provide on-site observation to check that appropriate materials are exposed in the footing excavation and to test the compaction of fill. These observations will allow us to compare the actual with the anticipated soil conditions and to check that the contractor's work conforms to the geotechnical aspects of the plans and specifications.

11.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on our interpretation of the observed conditions. If conditions other than those described in this report are encountered, our offices should be notified so that additional recommendations, if warranted, can be provided.

This report has been prepared for the exclusive use of Kennedy Jenks Consultants and their client North Coast County Water District for specific application to the proposed Gypsy Hill Tank Replacement in Pacifica, California as described herein. We cannot be responsible for the impacts of any changes in geotechnical engineering standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for un-consulted use of segregated portions of this report.

REFERENCES

Bonilla, M.G., 1971, Preliminary Geologic map of the San Francisco South Quadrangle and part of the Hunters Point Quadrangle, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-311.

AERIAL PHOTOGRAPHS

Date	Photo Number	Scale
03/11/05	KAV 9010-74-2, 3	1:10,000
8/15/00	AV 6600-2-16, 17	1:12,000
8/27/93	AV 4515-2-16, 17	1:12,000
07/1/91	AV 4075-3-23, 24	1:12,000
06/06/83	AV 2265-02-20, 21	1:12,000
04/28/75	AV 1188-02-22, 23	1:12,000
10/29/69	AV 933-03-19 , 20	1:12,000
04/23/58	AV 279-3-34, 35	1:7200
05/10/55	AV 170-3-26, 27	1:10,000
07/29/46	AV 9-12-2, 13-2	1:23,600

DISTRIBUTION

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Land/
Marine
Geotechnics

FIGURES

Source:
USGS Topographic Map, 1978

0 1000' 2000'
SCALE: 1" = 2000'

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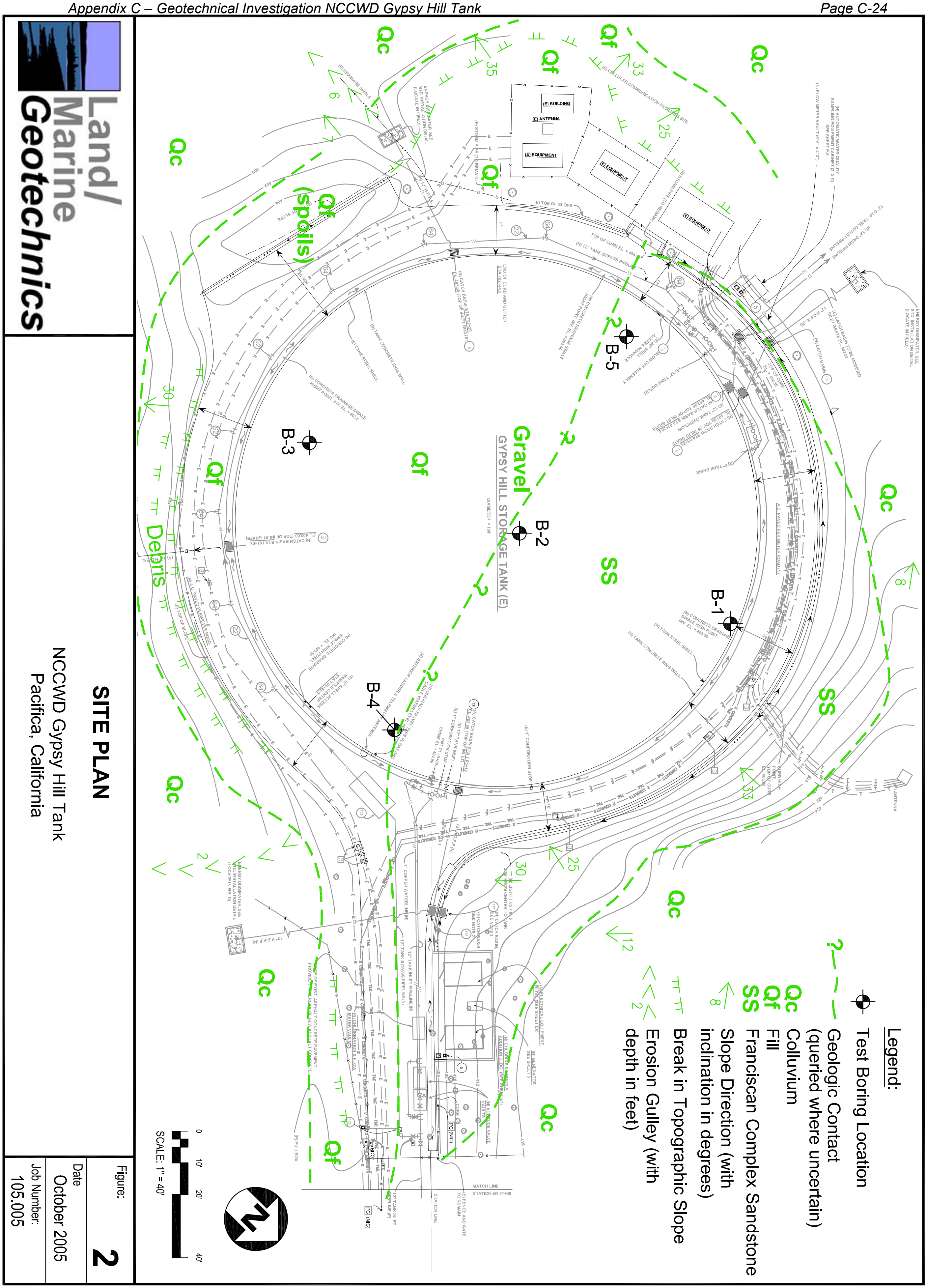
Land/Marine Geotechnics

SITE LOCATION MAP

NCCWD Gypsy Hill Tank
Pacifica, California

Figure:	1
Date	October 2005
Job Number:	105.005


Land/
Marine
Geotechnics
SITE PLAN
NCCWD Gypsy Hill Tank
Pacifica, California
Figure:
2
Date
October 2005
Job Number:
105.005
Legend:
Test Boring Location
Geologic Contact (queried where uncertain)
Qc Colluvium
Qf Fill
SS Franciscan Complex Sandstone
Slope Direction (with inclination in degrees)
Break in Topographic Slope
Erosion Gulley (with depth in feet)
SCALE: 1" = 40'
0
10'
20'
40'
GYPSY HILL STORAGE TANK (E)
DIAMETER = 168'
B-1
B-2
B-3
B-4
B-5
Qc
Qf
Qf (spoils)
SS
Gravel
Debris
A.C. PAVED PERIMETER ROAD (N)
MATCH LINE
STATION ER 01+30

APPENDIX A
FIELD INVESTIGATION

PROJECT: **NCCWD GYPSY HILL TANK** Pacifica, California

Log of Boring B1

Boring location: See Site Plan, Figure 2

Logged by: DWA

Date started: 9/2/05 | Date finished: 9/2/05

Drilling method: B-24 6-Inch Flight Auger

Hammer weight/drop: 140lbs./30-inches | Hammer type: Automatic

Surface Elevation: 404+/- feet

DEPTH (feet)	SAMPLE TYPE	Sample	SPT[1] N-Value	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				GP	GRAVEL (GP) gray, loose, dry, 3/4"-1" diameter						
1–8				CL/SC	Interbedded LEAN CLAY (CL) and CLAYEY SAND (SC) mottled yellow brown, brown, gray brown, and orange brown, stiff/medium dense, moist (Completely weathered claystone)						
3	MC		17		Liquid Limit = 27% Plasticity Index = 11%	UC		2,700		13.2	119
7	SPT		50								
8–14.5					SILTSTONE/CLAYSTONE yellow brown, intensely fractured, low hardness, friable to weak, deep weathering						
10.5	SPT		84								
14.5–15.5	SPT		52/6"		same as above, mottled brown, orange brown, and gray, highly to completely weathered, intensely fractured, weak to soft, plastic, some fine to medium sands						
15.5					EOH at 15.5 Feet						

1. Elevations based on plan titled "Existing Conditions at Gypsy Hill Storage Tank", undated.
2. Groundwater not encountered during drilling.
3. Boring backfilled with cement grout.

Project No.: 105.005

Figure: A-1

Land/ Marine Geotechnics

PROJECT: **NCCWD GYPSY HILL TANK** Pacifica, California

Log of Boring B2

Boring location: See Site Plan, Figure 2	Logged by: DWA
Date started: 9/2/05	Date finished: 9/2/05
Drilling method: B-24 6-Inch Flight Auger	
Hammer weight/drop: 140lbs./30-inches	Hammer type: Automatic

DEPTH (feet)	SAMPLE TYPE	Sample	SPT[1] N-Value	LITHOLOGY	MATERIAL DESCRIPTION (Surface Elevation: 404+/- feet)	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0				GP	GRAVEL (GP) gray, loose, dry, drain rock						
1				CL	LEAN SANDY CLAY (CL) mottled brown, dark brown and yellow brown, very stiff, moist, some fine to medium sands, few to little gravels up to 1/2", residual soil						
2	MC		38								
4					CLAYSTONE/SILTSTONE mottled orange brown, intensely fractured, soft, friable to weak, deep weathering						
5	SPT		69								
9	SPT		55/6"								
11.5					becomes low hardness						
14	SPT		56/6"								
14.5					EOH at 14.5 Feet						

1. Elevations based on plan titled "Existing Conditions at Gypsy Hill Storage Tank", undated.
2. Groundwater not encountered during drilling.
3. Boring backfilled with cement grout.

Project No.: 105.005

Figure: A-2

Land/Marine Geotechnics

PROJECT: **NCCWD GYPSY HILL TANK** Pacifica, California

Log of Boring B3

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2

Logged by: DWA

Date started: 9/2/05 | Date finished: 9/2/05

Drilling method: B-24 6-Inch Flight Auger

Hammer weight/drop: 140lbs./30-inches | Hammer type: Automatic

Surface Elevation: 404+/- feet

DEPTH (feet)	SAMPLE TYPE	Sample	SPT[1] N-Value	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1				GP	GRAVEL (GP) gray, loose, dry, gravels to 3/4"-1" diameter						
1–4				CL	GRAVELLY SANDY CLAY (CL) mottled brown, yellow brown, and dark brown, very stiff, moist, some fine sands, some angular to subangular gravels up to 1" diameter (Completely weathered bedrock)						
2	MC		39							9.5	115
4–13					SILTSTONE mottled yellow brown and dark brown, intensely fractured, low hardness, friable to weak, deep weathering						
5	SPT		30								
8	SPT		60/5"								
12	SPT		79								
13					EOH at 13 Feet						

1. Elevations based on plan titled "Existing Conditions at Gypsy Hill Storage Tank", undated.
2. Groundwater not encountered during drilling.
3. Boring backfilled with cement grout.

Project No.: 105.005

Figure: A-3

Land/Marine Geotechnics

PROJECT: **NCCWD**
GYPSY HILL TANK
Pacifica, California

Log of Boring B4

PAGE 1 OF 1

Boring location: See Site Plan, Figure 2
Logged by: DWA
Date started: 9/2/05 | Date finished: 9/2/05
Drilling method: B-24 6-Inch Flight Auger
Hammer weight/drop: 140lbs./30-inches | Hammer type: Automatic

LABORATORY TEST DATA

Surface Elevation: 404+/- feet

DEPTH (feet)	SAMPLE TYPE	SPT[1] N-Value	LITHOLOGY	MATERIAL DESCRIPTION	Type of Strength Test	Confining Pressure Lbs/Sq Ft	Shear Strength Lbs/Sq Ft	Fines %	Natural Moisture Content, %	Dry Density Lbs/Cu Ft
0–1			GP	GRAVEL (GP) gray, loose, dry, 3/4"-1" drain rock						
1–5			CL	LEAN SANDY CLAY (CL) mottled brown, yellow brown, and dark brown, stiff, moist, some fine sands, angular to subangular, gravels to 3/4" (Colluvium)						
3–4.5	MC	32								
5–14			CL	LEAN SANDY CLAY (CL) dark brown, very stiff, moist, some sand, angular, rock fragments to 3/4" (Colluvium) Liquid Limit = 29% Plasticity Index = 11%						
7–8.5	MC	53							9.1	117
12–13.5	SPT	65								
14–17.5			CL	LEAN SANDY CLAY (CL) mottled orange brown with gray veins, hard, moist, few pockets weathered claystone bedrock						
16–17.5	SPT	32								
17.5				EOH at 17.5 Feet						

1. Elevations based on plan titled "Existing Conditions at Gypsy Hill Storage Tank", undated.
2. Groundwater not encountered during drilling.
3. Boring backfilled with cement grout.

Project No.: 105.005
Figure: A-4

Land/Marine Geotechnics